Why do we not have a Consistent Design Method for Rubble Mound Breakwaters

On the Reliability of Rubble Mound Breakwater Design Parameters

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WHY DO WE NOT HAVE A CONSISTENT DESIGN METHOD FOR RUBBLE MOUND BREAKWATERS

(On the reliability of rubble mound breakwater design parameters)

1. INTRODUCTION

The title of my lecture might be a surprise to many professional engineers. Is it really possible that we do not have a consistent design method — after several hundred years of breakwater design and construction and also intensive research for the last 20 years? The answer is yes. The state of the art and the design tools are not satisfactory compared to those available in other branches of civil engineering such as for example structural engineering.

I shall try to explain the difficulties we are facing in breakwater engineering, especially for rubble mound breakwaters, by summarizing some of the uncertainties we have to deal with in the design process. A good overview of the uncertainties and the related consequences is of paramount importance to the designer. Without such knowledge it is impossible to evaluate the safety of a structure — a situation that is unacceptable for a professional engineer.

It is important to point out that the damage to a breakwater never depends on one single parameter such as for example the wave height. Moreover, the time history (duration) of the impact is of importance. This means that a discussion of uncertainties in breakwater design really should be based on the joint probability density functions of the involved parameters supplied with statistical information on the related persistance.

The following presentation is not in accordance with this since each parameter is treated separately. This is done for the sake of simplicity and also because it will still serve the main object of the presentation.

2. BASIC NEEDS IN DESIGN

For most civil engineering structures (buildings, bridges etc.) it is possible to design and check the structural performance by means of theory. This is because many years of research and experience have established the prerequisites which are

- *Information on size of all major types of loads*, often stated in standards as characteristic maximum and minimum values, which again are based on information of the statistical properties such as mean, standard deviation and frequency distribution.
- *Information on the structural response to the loads*, implemented in formulae which are in most cases the outcome of theories based on basic physics, but are in some cases more or less empirical.

Both loads and the response to those loads are known quantitatively to such an extent that meaningful safety factors can be specified in the various standards.

Although this is well known to all professional civil engineers, it is deliberately mentioned here as a reference for the following discussion on rubble mound breakwaters, for which the situation is completely different.

3. ENVIRONMENTAL LOADS

3.1 Waves

The ideal situation, depicted in Figure 1, where both short term and long term wave statistics can be established from on-the-site records almost never exists.

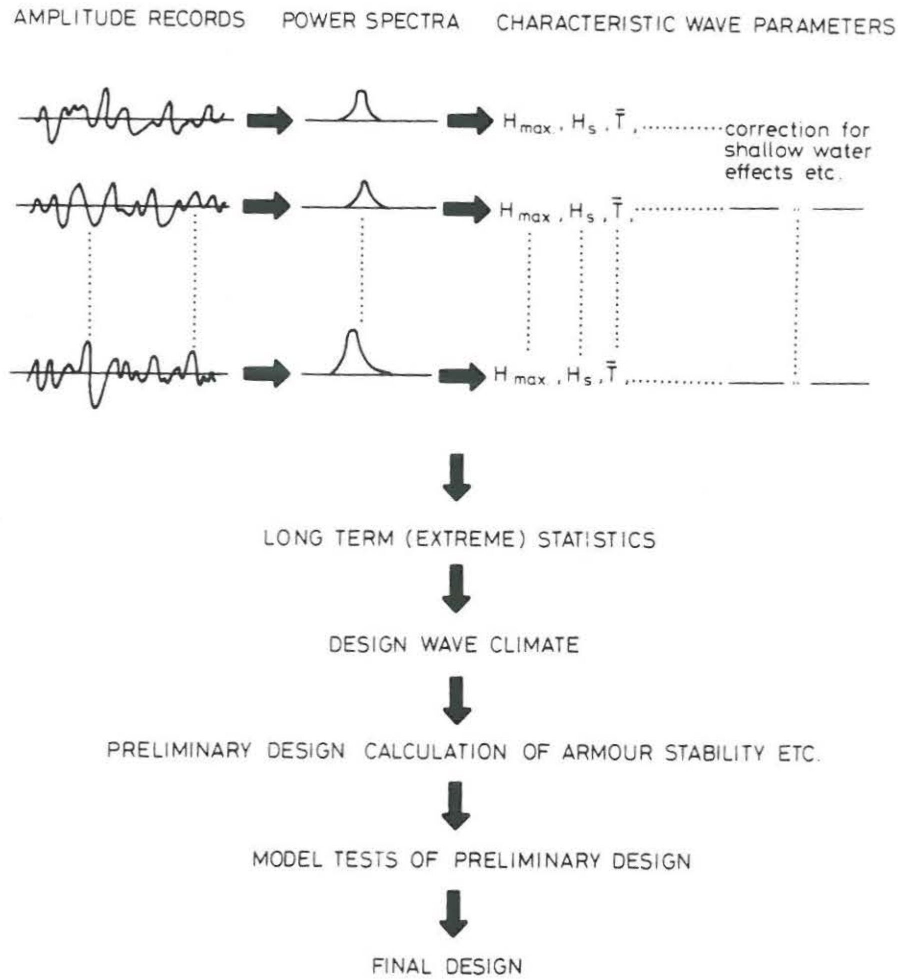


Figure 1. Ideal procedure for the establishment of design wave climate.

Usually one has to establish design wave conditions by hindcasting from meteorological observations and/or some wave records covering relatively short periods. In some areas visual wave observations from ships are available too.

It is clear from this that it is not possible to get reliable statistical values for all the wave parameters of importance. These are wave heights H, periods T, spectral shape, groupiness, direction of propagation and duration of storms.

Let us examine the wave height. This is generally the most important parameter since cover layer stability in terms of block weight is more or less proportional to the wave height in the third power. The uncertainties in the determination of extreme wave heights may result from the following sources:

A. Errors in measurements, visual observations or hindcasting of the wave data on which the extreme statistics are based.

B. Errors related to extrapolation from short samples to events of high return periods, i.e. low probability of occurrence.
Errors due to the choise of exceedence level.
Errors due to the method of fitting data to a chosen distribution.

C. Lack of knowledge about the underlying distribution for the extreme events.

D. Errors due to plotting positions.

E. Climatological variations.

ad A. Errors in wave data

Le Mehaute et al., 1984 discussed the uncertainties and systematic errors or bias related to the wave data under the assumption of errors being normally distributed. They reported the following "typical" normalized standard deviation σ' defined as the absolute standard variation divided by the expectation ("mean") value of H_s:

Direct wave measurement	$\sigma'_M = 0.05$	bias 0.00
Visual observations from ships	$\sigma'_M = 0.20$	bias 0.05
Hindcast (excluding hurricanes and other tropical storms)	$\sigma'_M = 0.15$	bias 0.05

It should be noted that the two last set of figures are applicable only when the sample populations are ranked statistically. A direct comparison in the time domain, i.e. comparison of individual sea states, generally shows larger discrepancies. Moreover the figures are average figures. For instance it is believed that wave data based on to-day's most advanced hindcast models applied to relatively restricted areas, such as the North Sea, where high quality weather maps are available, will show a smaller uncertainty.

ad B. Errors due to short samples.

Estimates on events of low probabilities are often performed in the following two different ways:

1) The extrapolation of data from frequent measurements or observations. The data are often compiled at intervals Δt = 3, 6 or 24 hours, which gives a large sample, N events, even in the case of a short time of observation or record length Y in years. The order of magnitude of N is often 1000 - 10,000.

2) The extrapolation of relatively few data sets representing the max significant wave height H_s for a number of storms exceeding a certain level, H'_s. The data are often determined from hindcasts and the sample size N is typically within the range 10 - 30.

Wang et al., 1983, examined the uncertainties related to the first method. They considered the long term distribution of H_s to be of the exponential type which also includes the often used Weibull distribution,

$$P(H_s) \equiv P[H \leqslant H_s] = 1 - \exp\left(-\left(\frac{H_s - A}{B}\right)^{\gamma}\right) \qquad (1)$$

where A is signifying the background noise level or lower-bound, B is the scale parameter and γ is the shape parameter. All three characteristic variables are normally determined by best fitting to the observed data.

Assuming the data asymptotically normally distributed about the underlying probability distribution function, eq (1), the authors obtained for large N the normalized standard deviation,

$$\sigma_s' \doteq \frac{1}{\gamma \ln(R\,\nu)} \left(\frac{R}{Y}\right)^{0.5} \tag{2}$$

where R is the return period in years, ν is the number of observations per year compiled at interval Δt and Y is the number of years of observations. Formula (2) is valid only for low probability levels and only for large samples $N = \nu Y$ of uncorrelated data. The latter implies that Δt should exceed approximately 24 hours, but because of little sensitivity on the confidence bands for H_s smaller values, as for example $\Delta t = 6$ hours, are often used.

It is stressed that the data to be used must belong to the same statistical population as the extreme events in question, i.e. wave data must be separated with respect to origin or type of waves, to directionality, to shoaling effects etc.

Example.

Taking R = 50 years, Y = 5 years, $\nu = 365$ observations per year and $\gamma = 1.2$
gives $\sigma_s' = 0.27$

Changing R and Y to 100 years and 3 years respectively
gives $\sigma_s' = 0.46$

The second method mentioned above is relevant to situations where data have to be obtained from hindcasting, which, due to the costs involved, restricts the number of data.

Rosbjerg, 1981, considered this case, where only maximum values η of H_s for independent storms exceeding a chosen level H_s' are taken into consideration, cf. figure 2.

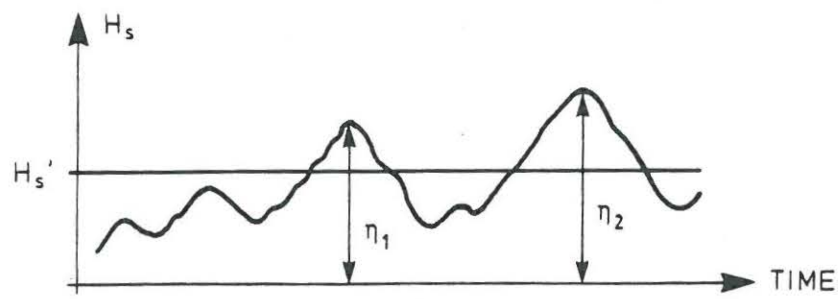


Figure 2. Data reduction by application of exceedence level, H_s'.

Rosbjerg assumed all the exceedences $\eta - H_s'$ to follow the exponential probability distribution,

$$P(H_s) \equiv P[\eta \leqslant H_s] = 1 - \exp\left(-\frac{H_s - H_s'}{\alpha}\right) \tag{3}$$

which is of the same type as the Weibull distribution, eq (1), with $\gamma = 1$.

The author also assumed the events η to occur at times corresponding to a Poisson-process with time dependent intensity. He arrived at the following expression for the R-year event defined as the value of η, which in average is exceeded once every R years,

$$H_s = H_s' + \alpha \ln \nu R \tag{4}$$

The corresponding absolute standard variation is

$$\sigma_s = \frac{\alpha}{(\nu\ Y)^{0.5}} \left(1 + (\ln \nu R)^2\right)^{0.5} \tag{5}$$

and the normalized standard deviation consequently

$$\sigma_s' = \frac{\sigma_s}{H_s} = \frac{\frac{\alpha}{(\nu Y)^{0.5}} \left(1 + (\ln \nu R)^2\right)^{0.5}}{H_s' + \alpha \ln \nu R} \tag{6}$$

The maximum likelihood estimate for α is

$$\hat{\alpha} = \bar{\eta} - H_s' \tag{7}$$

where $\bar{\eta}$ means average of η.

Nielsen et al., 1985, extended the analyses to include the Weibull distribution

$$P(H_s) = P[\eta \leqslant H_s] = 1 - \exp\left(-\left(\frac{H_s - H_s'}{\alpha}\right)^{\gamma}\right) \tag{8}$$

and found the following

$$H_s = H_s' + \alpha (\ln \nu R)^{1/\gamma} \tag{9}$$

$$\sigma_s = (\ln \nu R)^{1/\gamma - 1} \left[\frac{\alpha^2}{\gamma^2 \nu Y} + (\ln \nu R)^2 \frac{\alpha^2}{\nu Y} \left(\frac{\Gamma(1 + \frac{2}{\gamma})}{\Gamma^2(1 + \frac{1}{\gamma})} - 1 \right) + \frac{\alpha^2}{\gamma^4} (\ln \nu R) \cdot \ln(\ln \nu R))^2 \operatorname{Var}[\hat{\gamma}] \right]^{0.5} \tag{10}$$

ν is the average number of data per year and Γ the Gamma function.

The variance of $\hat{\gamma}$, $\operatorname{Var}[\hat{\gamma}]$, cannot easily be estimated, but by means of numerical simulation it is found that the term in (10) containing this quantity is highly dependent on the method for estimating the parameters in the Weibull distribution.

Petrauskas and Aagaard, 1971, found, by using a least square method, that the last term in (10) is insignificant. In this case the normalized standard deviation is

$$\sigma_s' = \frac{\sigma_s}{H_s} \cong \frac{(\ln \nu R)^{\frac{1}{\gamma} - 1} \left[\frac{\alpha^2}{\gamma^2 \nu Y} + (\ln \nu R)^2 \frac{\alpha^2}{\nu Y} \left(\frac{\Gamma(1 + \frac{2}{\gamma})}{\Gamma^2(1 + \frac{1}{\gamma})} - 1 \right) \right]}{H_s' + \alpha (\ln \nu R)^{1/\gamma}} \tag{11}$$

Nielsen et al., 1985, fitted the Weibull parameters by the method of moments, i.e. equating the first three moments of the distribution to those of the data, and found that the last term in (10) was of significance, namely in the order of 1/3 of the total standard deviation. The estimates on the parameter by the applied method of moments are given by

$$\frac{\Gamma(1 + \frac{3}{\hat{\gamma}}) - 3\Gamma(1 + \frac{2}{\hat{\gamma}})\,\Gamma(1 + \frac{1}{\hat{\gamma}}) + 2\Gamma^3(1 + \frac{1}{\hat{\gamma}})}{\left(\Gamma(1 + \frac{2}{\hat{\gamma}}) - \Gamma^2(1 + \frac{1}{\hat{\gamma}})\right)^{3/2}} = \frac{\overline{\eta^3} - 3\,\overline{\eta^2}\,\bar{\eta} + 2(\bar{\eta})^3}{(\overline{\eta^2} - (\bar{\eta})^2)^{3/2}} \tag{12}$$

$$\hat{\alpha}^2 = \frac{\overline{\eta^2} - (\bar{\eta})^2}{\Gamma(1+\frac{2}{\hat{\gamma}}) - \Gamma^2(1+\frac{1}{\hat{\gamma}})} \tag{13}$$

$$\hat{H}_s' = \bar{\eta} - \hat{\alpha}\,\Gamma\,(1-\frac{1}{\hat{\gamma}}) \tag{14}$$

$\overline{\eta^2}$ and $\overline{\eta^3}$ mean the average of sample values of η^2 and η^3, respectively, which are unbiased estimates of $E[\eta^2]$ and $E[\eta^3]$.

It should be noticed that the R-year event given by eqs (4) and (9) has a probability E of being equalled or exceeded in the specific lifetime L of the structure. For instance, if L is set equal to the return period R, this "encounter probability" E is as large as 63%. The relationship between R, L and E is given by

$$E = 1-(1-\frac{1}{R})^L \quad \text{or in case of R large} \quad R = -\frac{L}{\ln(1-E)} \tag{15}$$

For design purpose R in eqs (4) and (9) should be evaluated with respect to E and L through eq (15). For example in a 50 years lifetime there is a 10% probability that the structure is hit by the 500 years' return period storm.

Eqs (6) and (11) make it possible to determine the necessary sample length when a prediction for a given return period with a prescribed accuracy and confidence is required. Following the normal distribution the products of σ_s' with 0.84, 1.28, 1.65 and 2.32 define the upper bound of spread corresponding to a confidence level of 80%, 90%, 95%, and 99%, respectively. For instance, the prediction of an event with 90% confidence and an uncertainty of no more than 0.20 imply that 1.28 $\sigma_s' \leq 0.20$. Inserting this in eqs (6) or (11) gives the corresponding number of years of observation Y for given ν and R.

Example.
The accuracy of estimates based on a restricted number of hindcasted data sets might be illustrated by the following example. The Delft Hydraulics Laboratory did a hindcast study for a specific deep water location in the Mediterranean Sea and found for a 20 years period the following 17 most severe storms, Table 1:

Table 1. Example of hindcasted storm wave data for a 20 years' period.

Rank i	Max H_s (= η) meters	Peak period T_p seconds	Average wave direction degrees
1	9.32	14.0	143
2	8.11	14.1	139
3	7.19	13.4	123
4	7.06	10.8	123
5	6.37	11.9	143
6	6.15	11.1	185
7	6.03	12.3	135
8	5.72	10.5	176
9	4.92	10.7	150
10	4.90	10.6	129
11	4.78	11.8	161
12	4.67	9.9	120
13	4.64	9.2	122
14	4.19	10.5	137
15	3.06	11.1	154
16	2.73	8.2	153
17	2.33	8.3	126

If we choose $H'_s = 4.0$ m we find $N = 14$ storms exceeding this level over a period $Y = 20$ years, which gives $\nu = 14/20$. According to eq (7) α can be estimated to $\hat{\alpha} = 2.00$ m. It can now be tested if the data follow the assessed distribution, for example the exponential type given by eq (3). In this case a straight line with slope 1:1 should be obtained by plotting $\eta_i - H'_s$ against $-\hat{\alpha}\ln(1 - P(\hat{\eta}_i))$, where $P(\hat{\eta}_i) = 1 - \frac{i}{N+1}$, (Gumbel plotting positions). Figure 3 shows that the fit is reasonable.

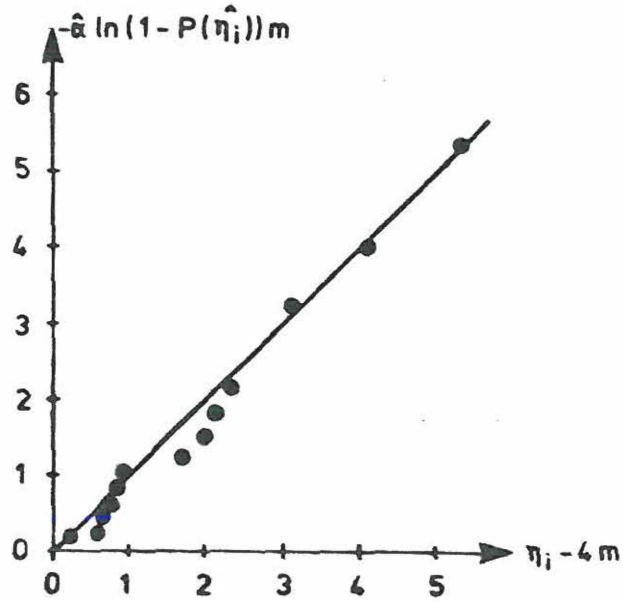


Figure 3. Test on exponential distribution of wave height exceedences.

Formulae (4) - (6) are then valid and the expectation values and the standard deviations can be calculated for various return periods, for instance

Return period R years	H_s meters	σ_s meters	σ'_s
50	11.11	1.97	0.18
100	12.50	2.33	0.19

Note that a change in H'_s for example to 3.50 m, which still gives $N = 14$, will change H_s and σ_s! This important problem is not discussed further here.

It is obvious that the 14 data points also fit a Weibull distribution.

If all the 17 data points given in Table 1 are considered, it corresponds to a exceedence level of $H'_s \cong 2.25$ m because the lowest value in the data set is $H_s = 2.33$ m. It turns out that in this case the data do not fit neither the exponential distribution, eq (13), nor the Weibull distribution, eq (8). However, if the exceedence level is not interpreted as the physically true cut-off level, but is regarded a fitting coefficient only, like α and γ, then the 17 data points follow the Weibull distribution very closely, as demonstrated in Figure 4. The coefficients are in this case $H_s' = 0.73$ m, $\alpha = 5.27$ m and $\gamma = 2.80$, all estimated by the method of moments.

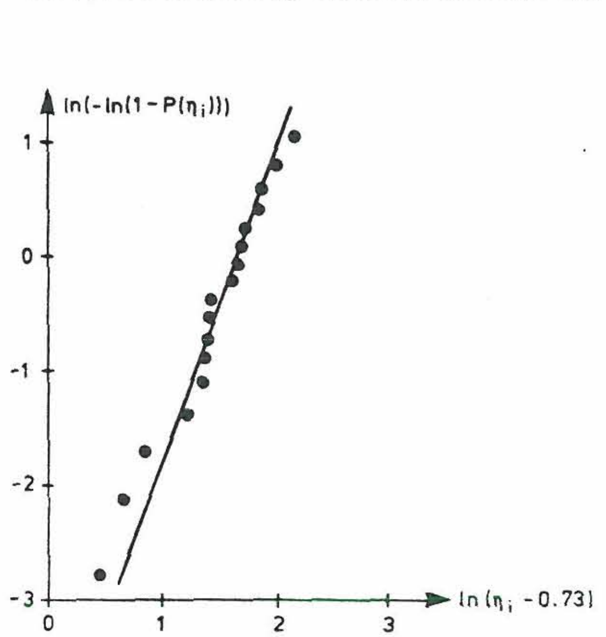


Figure 4. Data fit to the Weibull distribution. Gumbel plotting positions.

From eqs (9) - (11) we obtain the following corresponding values

Return period R years	H_s meters	σ_s meters	σ_s'
50	9.19	0.88	0.10
100	9.71	0.97	0.10

The Weibull distribution shown by the straight line in Figure 4 is a result of the chosen method of fitting. A least square fit or a visual fit will produce different lines and different estimates on the extreme events.

ad C. and D. Errors due to the lack of knowledge on the true long term distribution and due to plotting positions.

Several probability distributions are used to describe extreme wave height statistics. These include for example the log-normal distribution, the extremal type I or Gumbel or Fisher-Tippett I distribution, the extremal type II or Fretchet or Fisher-Tippett II distribution, the Ward-Borgman distribution and the extremal type III or Weibull distribution. Although each of these distributions has a theoretical base, they cannot be evaluated and related to the extreme waves on a physical base. As a consequence they are only fit to the available data. Most often the scales used for the plotting are such that the chosen distribution lies on a straight line, simply because of the more convenient visualization of the extrapolation. However, when extrapolating, one should always be aware of possible physical processes, such as for example wave breaking, which might interrupt the probability distribution at some probability level.

It follows from these comments that due to unknown extreme distribution errors can only be estimated by a sensitivity analysis in which various distributions are fitted. Table 2 shows such an analysis by the Delft Hydraulics Laboratory performed on the wave data given in Table 1.

Table 2. Example of influence of choise of extremal distribution and plotting position on low - probability wave heights. Data by Delft Hydraulics Laboratory.

Extremal distribution	Plotting position	Correlation coefficient	Return period H_s 50 year	100 year
Type I Gumbel	Gumbel	0.9875	11.0 m	12.2 m
	Gringorten	0.9852	10.3 m	11.3 m
Ward/Borgman	Gumbel	0.9872	9.8 m	10.5 m
	Gringorten	0.9920	9.4 m	10.1 m
Type III Weibull	Gumbel	0.9877	9.6 m	10.2 m
	Gringorten	0.9877	9.3 m	9.9 m

Although no accurate figures can be given it seems reasonable from this table and the above given example based on the distribution, eq (3), that due to unknown extreme distribution a normalized standard deviation σ'_D might be in the order of

$$\sigma'_D \cong 0.05 - 0.10.$$

In order to plot the data a position formula must be adopted. Many different plotting positions, all based on some statistical considerations, exist, but it is not easy or possible to select a specific one as the most correct. For this reason it is reasonable to estimate the error due to plotting positions by sensitivity analyses involving a number of reasonable plotting rules.

Table 2 gives an example where only two plotting rules are used, namely

Gumbel/Weibull
$$P(\eta_i) = 1 - \frac{i}{N+1} \tag{16}$$

and

Gringorten
$$P(\eta_i) = 1 - \frac{i - 0.44}{N + 0.12} \tag{17}$$

It is seen that significant deviations in the estimated extreme wave height occur due to the plotting rules. It is believed that a realistic normalized standard deviation σ'_p on extreme events will be in the order of

$$\sigma'_p \cong 0.05$$

ad E. Errors due to climatological variations.

An additional source of uncertainty is the natural variation of the wave climate. Le Mehaute et al., 1984, considered this difficult problem under the assumption of the natural climatology being ergotic and stationary and governed by the statistical law of Weibull distribution. By setting Y = R in eq (2) they found that the normalized standard deviation of climatological variations in R years at a particular location is given by

$$\sigma'_C = \frac{1}{\gamma \ln(R\nu)} \tag{18}$$

If we for instance estimate $\gamma \cong 1.2$ as proposed by the authors we find for $\nu = 365$ and R = 50 or 100 years $\sigma'_C \cong 0.08$.

Combined errors.

The above mentioned sources of uncertainty can be assumed mutual independent except for an unknown but probably weak correlation between the climatological variation and the data samples.

The total normalized standard deviation might then be estimated by

$$\sigma' \cong (\sigma_M'^2 + \sigma_s'^2 + \sigma_D'^2 + \sigma_p'^2 + \sigma_C'^2)^{0.5} \tag{19}$$

With reference to the foregoing discussion one can establish the following two examples:

Examples.

Direct wave height measurement. ν = 365 observations per year. Y = 5 years. R = 50 years.

$$\sigma' \cong (0.05^2 + 0.27^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.30$$

Hindcasted wave heights. 14 data sets over Y = 20 years. R = 50 years.

$$\sigma' \cong (0.15^2 + 0.18^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.26$$

From this it is seen that, even with what is generally regarded reasonable lengths of data sample and observation period, the uncertainty related to the 50 year event is significant and in the order of $\sigma' \cong 0.25 - 0.30$. If we assume normally distributed random variables it means a 16% probability of the wave height being bigger than 1.25 - 1.30 times the estimated height.

The uncertainty increases significantly when the lengths of data sample and the period of observation are reduced to figures below those given above.

The difficulties in obtaining reliable estimates on design wave heights might also be illustrated by the following example from the Norwegian Ekofisk North Sea offshore field given by Professor Tørum of Norway.

In 1970 the 100 year design wave height was estimated to be 19.6 m. In 1972 it was 23.6 m; in 1977 28.0 m; in 1981 up to 34 m. And finally in 1984 the estimate was 28 m with an uncertainty of approximately ± 15%! This big uncertainty exists despite the large resources spent on wave recordings, wave statistics etc. in this prospective offshore area. These resources are much larger than those available for the design of breakwaters.

It is not only the wave height that is of importance but also

- the wave period
- the spectral shape
- the horizontal, directional spread of the wave energy / short crestedness of the waves
- the groupiness
- the direction of the propagation
- the duration / time history of the storms

Therefore the uncertainty related to the estimation of these parameters should also be evaluated. It takes a lot of work and research to perform such an analysis, also because generally it is the reliability of the "joint parameters" which are of interest. This problem is not discussed further here. However, it is obvious that it all adds to the uncertainty on design wave climate estimations.

If the breakwater is in "shallow-water" and the wave data are from an offshore location then we have to include the uncertainty related to shallow water effects such as:

- Refraction, i.e. change of wave direction and wave height due to oblique wave approach.
- Shoaling, i.e. change of wave height and wave length due to water depth variations perpendicular to the coast.
- Wave breaking, due to instability by decreasing water depth.
- Wave set-up, i.e. change of the mean water level due to changes of the wave radiation stress.

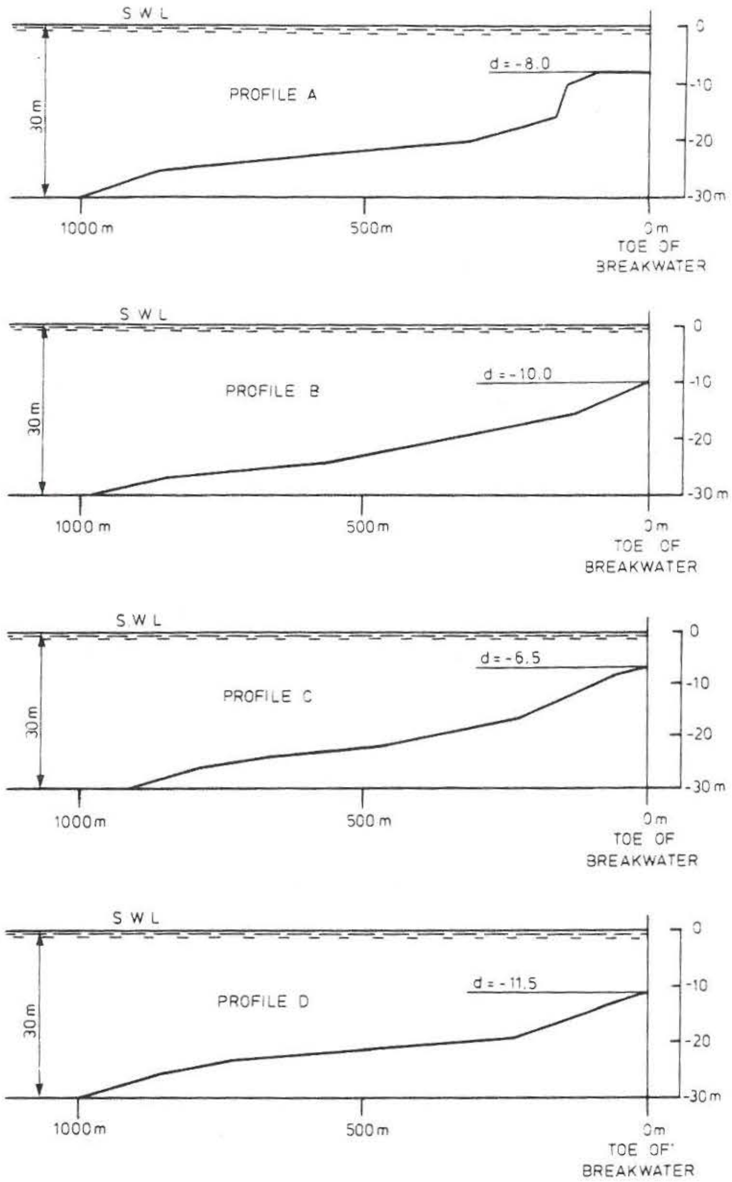
Besides these effects we also have:

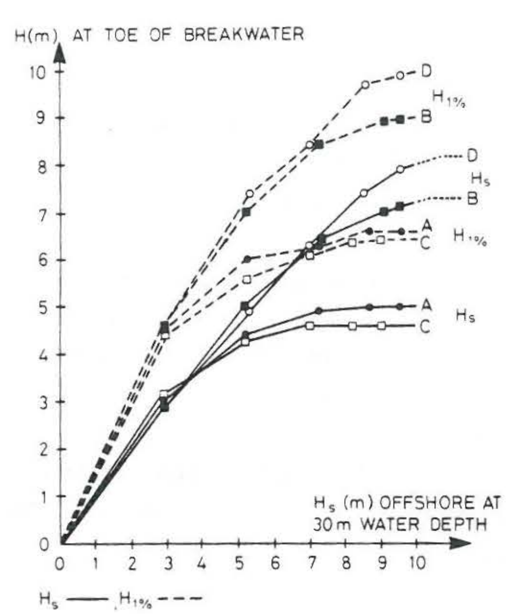
- Tidal water level variations.
- Barometric pressure variations.
- Wind set-up, i.e. wind induced change of the mean water level.
- Seiches.
- Currents.

The uncertainties related to all these parameters or phenomena are in general not well established except for tidal water level variations. Consequently a quantitive discussion on uncertainties is not possible. However, in the next paragraph we shall evaluate the importance of reliable data by a sensitivity analysis of the structural response to some of the parameters.

It has often been pointed out that estimates on design waves are much more reliable in shallow water than in deep water due to the depth limited wave heights. This is true but it should be mentioned that no wave theory exists which can predict with good accuracy the absolute wave height distribution and maximum wave heights in shallow water with breaking waves. Moreover, it should not be overlooked that the water level is also a very important parameter when breakwaters are designed for a certain amount of overtopping.

Another point which should be stressed is the sensitivity of shoaling/wave breaking to variations in the sea bed profile. This is illustrated in Figure 5, where the wave heights of the incoming waves at the toe of a breakwater are determined for four different foreshore bottom profiles. The breaker index $\gamma_{H_s^{max}}$, defined as the ratio of the max significant wave height, H_s^{max} and the water depth, d at the toe, is also given in the figure together with the breaker index $\gamma_{H_{1\%}^{max}}$ related to the maximum value of wave heights, exceeded by 1% of the waves.





PROFILE	BREAKER INDEX $\gamma = \frac{H^{max}}{d}$ AT TOE OF BREAKWATER	
	$\gamma_{H_s^{max}}$	$\gamma_{H_{1\%}^{max}}$
A	0.63	0.83
B	0.73	0.90
C	0.71	0.98
D	0.70	0.87

Figure 5. Example of sensitivity of depth limited wave heights to differences in foreshore bottom profiles. Delft Hydraulics Laboratory.

The wave heights was determined by DHL in wind-wave flume model tests without the breakwater.

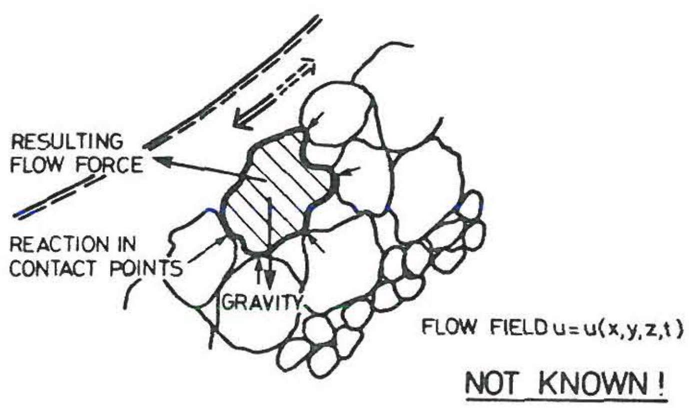
It is seen that a good estimate on the wave height in front of a breakwater in shallow water must be based on model tests with a correct reproduction of the foreshore topography. This means that in case of significantly varying bottom topography along the breakwater it is necessary either to test many sections or preferably to test the hole structure in a three-dimensional model.

4. SENSITIVITY IN STRUCTURAL RESPONSE TO THE ENVIRONMENTAL LOADS

The following is not intended to be a complete discussion as only a few, but important, problems will be discussed.

4.1 Hydraulic stability of the armour layer

The difficulties related to a purely theoretical stability analysis might be illustrated by considering the forces on an armour unit, see Figure 6.



GRAVITY : $F_G = g \rho_w \left(\frac{\rho_s}{\rho_w} - 1\right) d^3$

FORM DRAG: $F_{D,F} = C_F \, \rho_w \, d^2 |u| u$

SURFACE DRAG: $F_{D,S} = C_S \, \rho_w \, d^2 |u| u$

LIFT: $F_L = C_L \, \rho_w \, d^2 u^2$

INERTIA, FROUDE-KRYLOV: $F_I = C_I \, \rho_w \, d^3 u'$ (pressure grad. undisturb. flow)

INERTIA, ADD. HYDRODYN. MASS: $F_H = C_M \, \rho_w \, d^3 u'$ (change of flow field by the body)

COEFFICIENTS C are functions of Keulegan-Carpenter No. and Re No. and will vary considerably in time.

Figure 6. Forces on armour unit.

As a consequence stability formulae are semiempirical and formulated as an equality between a characteristic drag flow force and the stabilizing gravity force multiplied by unknown functions to take care of slope angle, friction, interlocking, wave period etc.

The various formulae show that the stability in terms of required mass, M of the armour unit is more or less proportional to the wave height in the third power. This very strong dependence put emphasis on the need for precise estimates on wave heights. This is depicted in Figure 7, where the relative variation of M in the range H ± σ (H) is shown. σ (H) is taken as 0.3 $\hat{H}$, cf. paragraph 3.

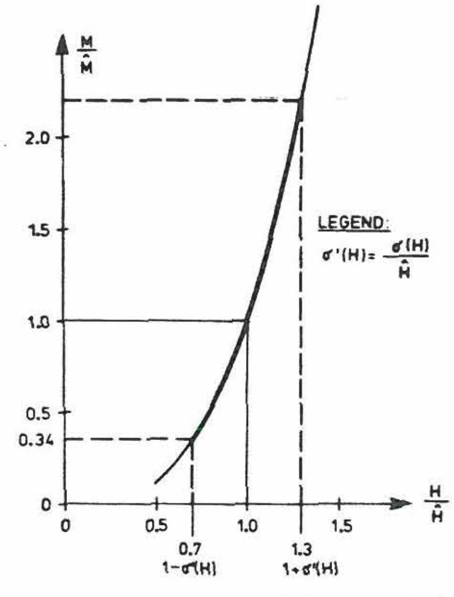


Figure 7.
Influence of wave height, H on required mass, M of armour unit.

The armour layer stability is also affected by the wave period T, but the variation with T is generally found to be much weaker than the variation with H. However, Gravesen et al. 1979 found a strong influence and proposed that the wave period is taken into consideration by using H_s^2 L_p in the stability formulae instead of H_s^3. L_p is here the wave length corresponding to the spectral peak period T_p. As L_p is more or less proportional to T_p^2 this implies a dependence of M on T_p as schematized in Figure 8. As a characteristic standard deviation is chosen $\sigma(T_p) = 0.15\ \hat{T}_p$.

Gravesen et al.'s findings are related to an armour layer of cubes with slope 1:2 but surmounted by a vertical wave wall, which affects the stability in the case of larger wave.

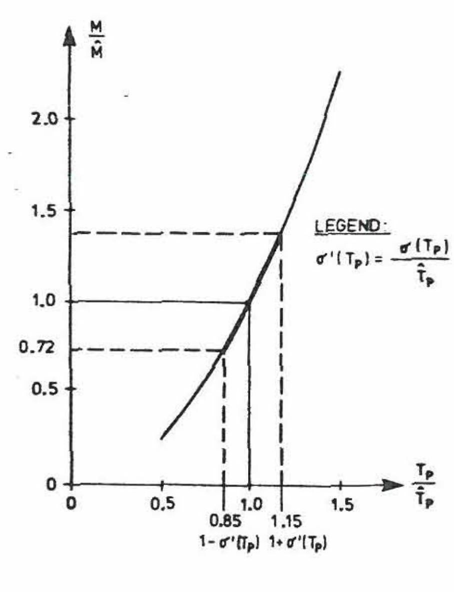


Figure 8.
Influence of peak period on required mass, M of armour unit as proposed by Gravesen et al., 1979.

A somewhat weaker but still significant dependence was found by Burcharth, 1979 in stability tests with Dolosse armour exposed to regular waves, Figure 9. The same reference also shows that run-up increases significantly with the wave period.

Figure 10 shows a replot of stability tests in regular waves with uniform stones, Dai & Kamel, 1969 and rip-rap, Ahrens, 1975. It is seen that the stability sensitivity to wave period is small in the case of uniform stones and large, but with opposite trends, for rip-rap.

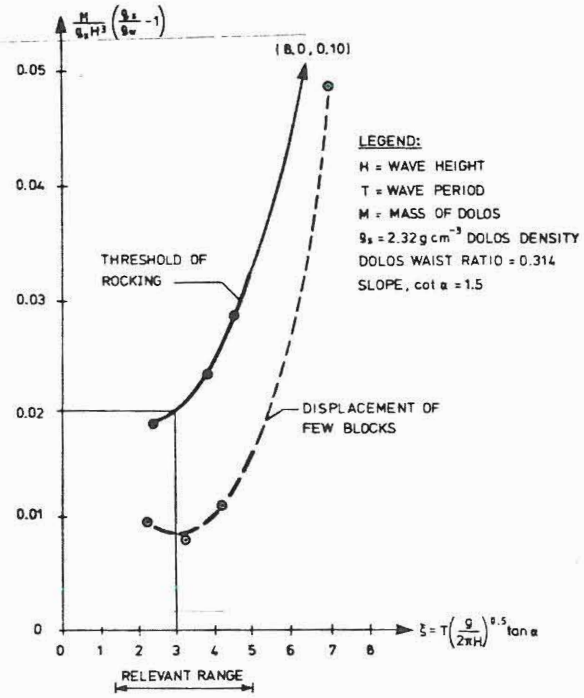


Figure 9.
Example of influence of wave periode on required mass of Dolosse armour units. Tests in regular waves. Burcharth, 1979.

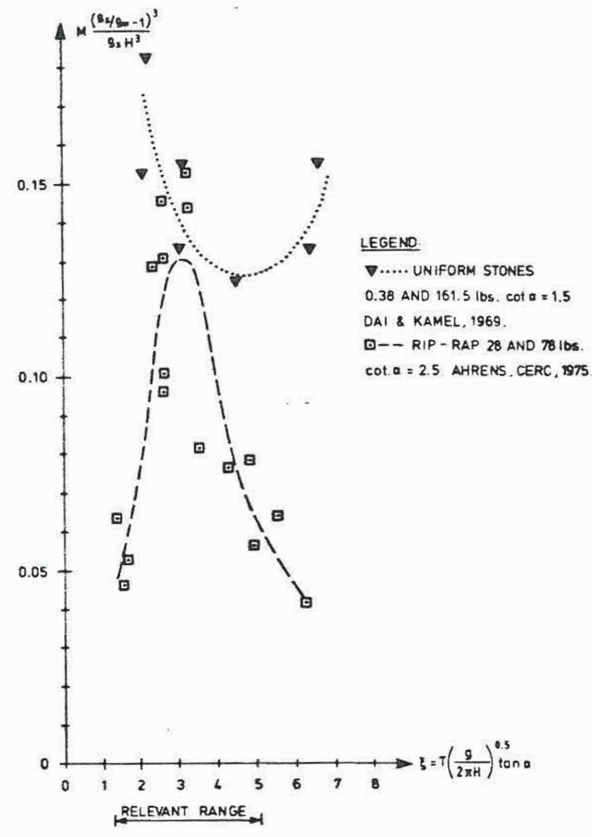


Figure 10.
Examples of influence of wave period on required mass of uniform armour stones and rip-rap. Replot by Dai & Kamel, 1969, and Ahrens, 1975.

In Figure 11 the data are normalized with respect to $\zeta = 3$ for easy mutual comparison of the wave period sensitivity. $\zeta = 3$ is a characteristic average value for rubble mound breakwater design wave situations. It is seen that an uncertainty on T (for example $\sigma'(T) = 0.15$) around the value $T_{\zeta=3}$ only gives relatively small variations on the required mass M.

This is somewhat contradictory to Figure 8 but might be explained by the influence of the wave wall as explained above.

Figure 11 also shows that the larger the porosity of the armour layer the more vulnerable the armour is to large wave periods (Dolos armour has the largest porosity and rip-rap the smallest). This is due to the "reservoir effect" of the pores as explained in Burcharth et al., 1983. A stability minimum seems only present for the relative impermeable rip-rap.

Note that the data in the Figures 9, 10 and 11 are from tests with regular waves.

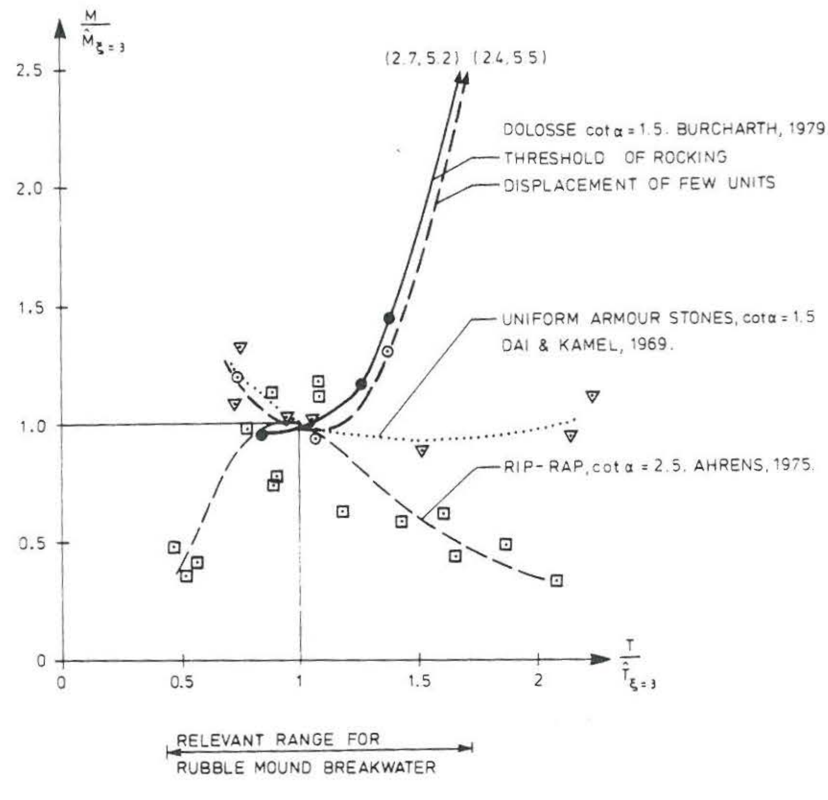


Figure 11. Example of influence of wave period on required mass of armour units and rip-rap. Regular waves. Data normalized with respect to the estimated values $\hat{T}_{\zeta=3}$ *and* $\hat{M}_{\zeta=3}$ *corresponding to* $\zeta = T\left(\frac{g}{2\pi H}\right)^{0.5} \tan\alpha = 3$.

The examples show that the effect of the wave period on armour stability is not clarified in general.

4.2 OVERTOPPING

The design conditions are often related to overtopping of the breakwater. This is the case where roads, reclaimed areas, berths, installations etc. are located behind and close to the breakwater.

Overtopping is very sensitive to variations in wave height and mean water level. Besides this also variations in wave period, wave direction and wave shortcrestedness affect the overtopping.

The sensitivity to the wave height can be illustrated by the example given in Figure 12, which shows some scale model test results from a rubble mound breakwater with a wave wall.

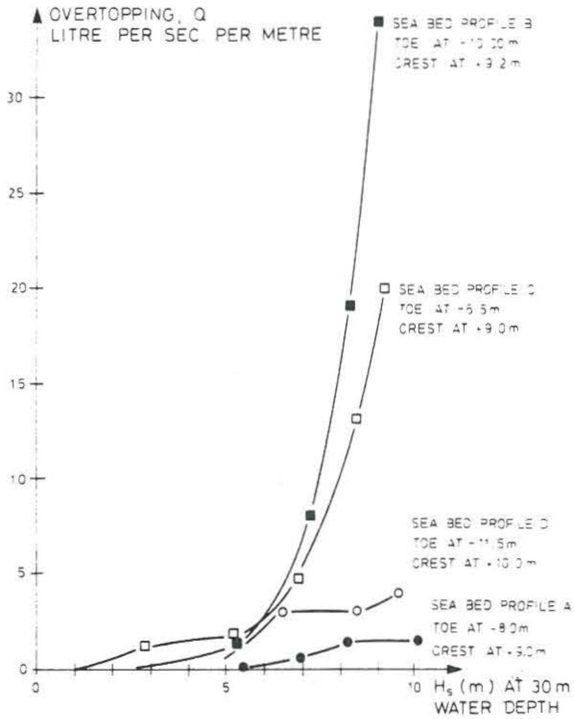


Figure 12.
Example of sensitivity of overtopping to wave height. Delft Hydraulics Laboratory. Sea bed profiles refer to Figure 5.

It is seen that the overtopping, Q increases exponentially when the wave height exceeds a certain value. A 10% increase in H_s can easily cause doubling of Q. The exponential growth of Q with H_s usually makes $\log Q$ a linear function of H_s.

Based on different scale model experiments Jensen et al., 1979, presented a more general description by means of the parameters QT_z/B^{*2} and $H_s/\Delta h$. T_z is mean zero crossing wave period, B^* is a representative horizontal dimension and Δh is the vertical distance from still water level to the top of the crest or wave wall. By introducing Δh also the influence of water level is taken into account. Figure 13 shows an example given by Jensen et al.

Figure 13 clearly shows that even small variations in the still water level might cause significant variations in overtopping.

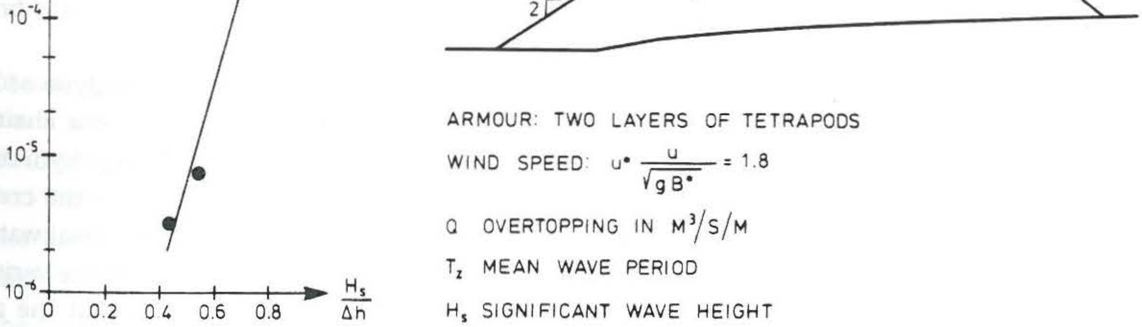


Figure 13. Example of sensitivity of overtopping to wave height and still water level. Shallow water conditions. Jensen et al., 1979.

4.3 Directionality of the waves

Until to day nearly all breakwater model tests have been performed with uni-directional (2-D, long crested) waves. However, in nature the waves are directional (3-D, short crested) with horizontal spread of energy.

It is generally believed that 2-D waves is a good approximation to natural waves in shallow water due to the refraction which tends to make the waves long crested. However, Thunbo et al., 1984, found from a scale model experiment with a stone armoured breakwater with slope 1:2 in shallow water that 2-D waves caused 30-50% more damage than 3-D waves. This compares approximately to the necessity of a 40% increase in armour stone weight when going from 3-D waves to 2-D waves at the same damage level. Figure 14 shows some of the results.

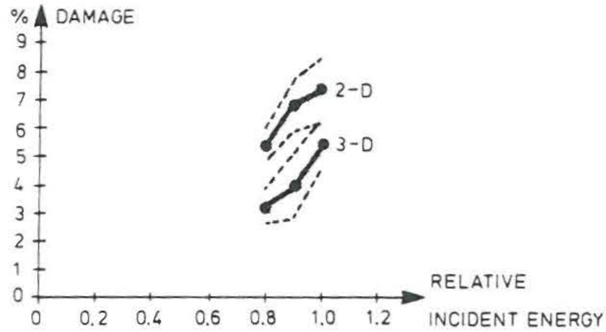


Figure 14. Example of comparison of 2-D and 3-D wave effects on stone rubble mound breakwater. Thunbo et al., 1984.

Shutler of HR, Wallingford reported from similar tests that no significant difference in 2-D and 3-D waves were found (scatter in the test results blurred possible differences).

It is concluded that there is still great uncertainty about the effect of wave directionality.

5. MODEL TESTS

Model tests are still necessary for practically all breakwater designs that depart from the very simple ideal design often tested in basic model studies of armour stability.

The reliability of model tests is therefore a question of great importance.

5.1 Reproduction of waves and data processing

The first point to discuss is the uncertainty related to the generation and analysis of laboratory waves. This problem was investigated by an IAHR Working Group, which was chaired by Joe Ploeg of Canada. The group consisted of representatives from some of the large hydraulic laboratories. Each laboratory performed the same experiment on a breakwater with the crest at MWL and exposed to some pre-specified waves. The wave climate in front of the breakwater and the water level variations behind it were recorded and analyzed. The results from the various laboratories deviated significantly and it was only after a great deal of thought that the reasons for these variations were explained. It turned out that the discrepancies to a great extent were due to differencies in the processing of the recorded data.

5.2 Scatter in test data

Another problem in model testing is the scatter in the test data signifying the response to the waves. This can be illustrated by some stability tests performed at the University of Aalborg with a Dolosse armour layer having a slope of 1 in 1.5 and exposed to irregular waves. For each of five different significant wave heights, H_s, 15 tests with identical wave trains were run with the object of studying the movements in terms of rocking and displacement of Dolosse. Very careful visual observations were made simultaneously by four people each covering a small area. A mirror system was used to obtain reliable observations in the splash and underwater zones. Each test was run for 20 minutes corresponding to approximately 1200 waves.

Some test results are shown in Figure 15, which illustrates the observed scatter related to the number of rocking and displaced blocks. These two modes of movement are relevant to the mechanical integrity of the blocks and the hydraulic stability of the armour layer.

Although direct recording of stresses in and/or recording of speed/acceleration of the blocks are much better than visual observations, the diagrams clearly illustrate the fact that reliable estimates of stability can be obtained only when tests are repeated several times. This is a fact which should not be overlooked.

It means that it might be necessary to apply a large safety factor if only a few tests are carried out, or to spend a lot more money performing many more tests than is normally the case at the moment. This is especially true for the complex, fragile types of armour units since it is seen from the Figure that the normalized standard deviation σ/μ for the numbers of displaced units is very large for small degrees of movements or damage corresponding to the design criteria for such units.

For large degrees of damage, i.e. failure situations, the scatter is reduced.

It should be mentioned that separation of rocking and of displacement in the "two" diagrams is not entirely meaningful and should be avoided in design diagrams. It is also important to remember that the scatter (e.g. in terms of the standard deviation) is dependent on the size of the test section.

5.3 Scale effects

The reliability of breakwater scale models has often been and still is seriously questioned and in most cases exclusively with reference to scale effects (thus forgetting the afore mentioned points). All scale models involve improper representation of some forces simply because only two types of forces at a time can be represented to scale. Therefore the question is "how much" is the model biased.

The two dominating forces in wave action models are gravity and inertia forces. Considering only these two types of forces the Froudian model scale law used for breakwater models ensures dynamic and kinematic similarity of the scale model and the prototype. Consequently viscous forces and surface tension are not reproduced to scale.

Viscous effects

For a wave exposed breakwater the flow is extremely unsteady. In some parts of the porous structure the flow will be turbulent or laminar all the time but in some part intermittent between the two flow-regimes, as discussed by Burcharth 1983.

The turbulent dragforces will scale like the inertia forces, because the viscous contribution is insignificant.

The flow-regime in granular structures is usually characterized by a Reynolds' number defined as

$$\mathbb{R} = \frac{V\,d}{\nu} \quad , \tag{20}$$

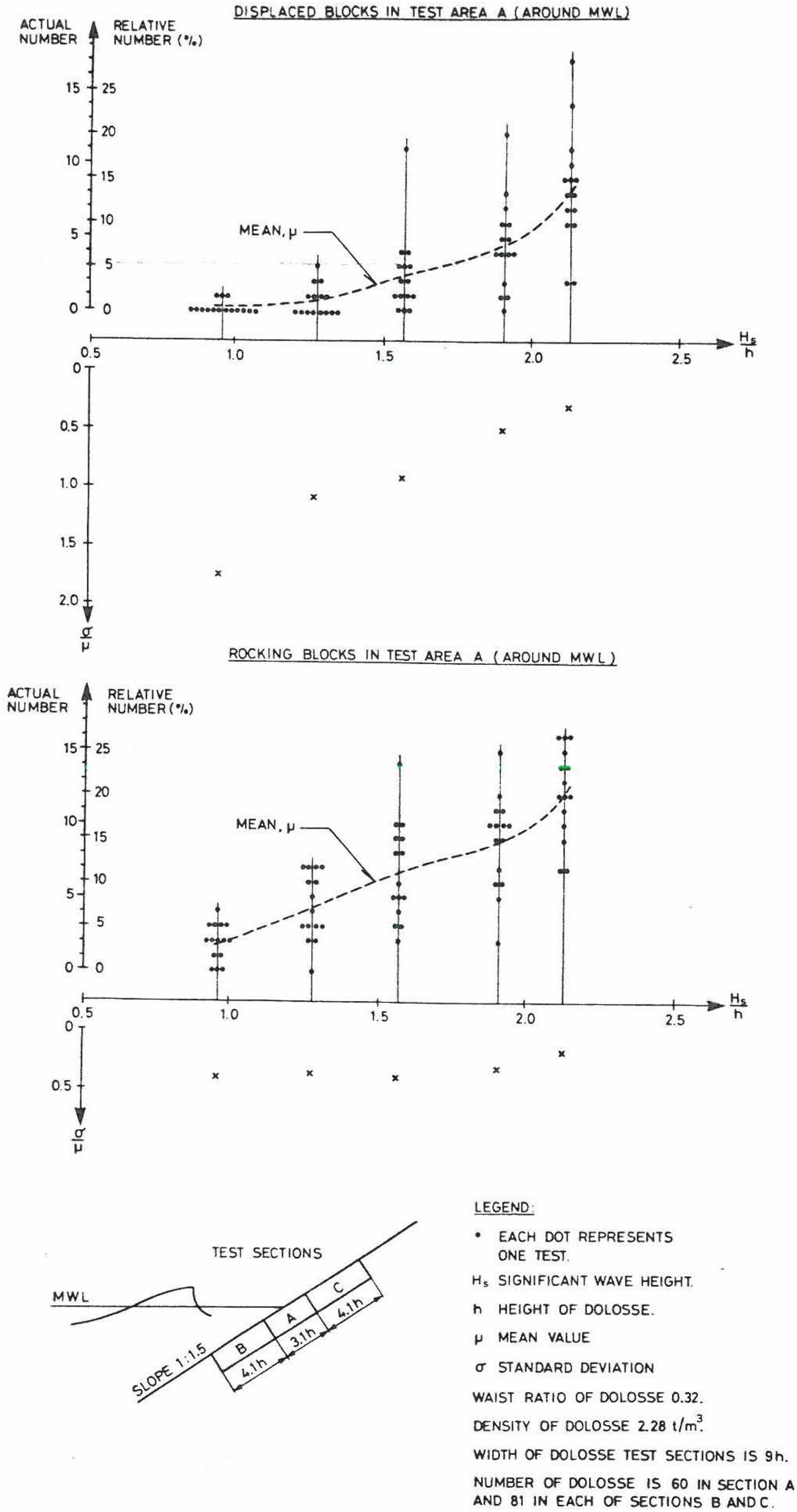


Figure 15. Example of scatter in armour stability tests.

where V is a characteristic flow velocity, d is a characteristic length and ν the kinematic viscosity of the liquid. When evaluating the unsteady flow in breakwaters it has become a tradition to use a constant figure for V which, more or less, is the maximum particle velocity of the incoming wave, i.e. $V \equiv (g\,H)^{0.5}$, where g is the gravitational constant and H is the wave height. d is usually taken as a typical diameter of the armour units/filter layer stones/core material, thus characterizing the width of the flow channels.

The primitiveness of this approach is obvious, but it is difficult to come up with an alternative which is both meaningful and simple.

Many researchers have studied viscous scale effects in breakwater models and the state of the art might be summarized as follows:

- No "significant" scale effect is observed in the "hydraulic stability" of the armour layer if $\mathbb{R} \geq 1 - 3 \cdot 10^4$ (d being a characteristic diameter of the armour units) and if the filter stones and the core material are geometrically to scale.

  However, it is important to notice that this statement is conclusive only in relation to mechanically strong armour units such as for example natural stones and concrete cubes. For the more fragile, complex types such as Dolosse and Tetrapods a scale effect which is not identified from visual observations of armour unit movements in the model might, when transferred to prototype, cause a very different amount of breakage. Timco et al., 1984, investigated this in some tests with Dolosse units with correctly scaled mechanical properties. They found that the influence of core permeability on the breakage of the Dolosse was very dependent on the geometrical scale.
- Run-up and overtopping are affected also by the porosity of the filter layer and the core. It has not been properly investigated how much changes in the size of the stones in order to obey the Reynolds' criteria stated above will bias run-up and overtopping.
- The reflection of waves from a breakwater scale model is practically independent of the permeability of the core, Timco et al. 1984.
- There is evidence that ultimate failures of rubble mound structures armoured with strong units can be studied with great accuracy in scale models. This statement is mainly based on a comparative study by DHI, Jensen et al., 1985, of the failure of the Thorshavn breakwater in the Faroe islands. This study is significant because of the availability of the prototype records of the waves in front of the breakwater throughout the damaging storm. The Reynolds' numbers in the model were about $4 \cdot 10^4$ for the armour stones and about $5 \cdot 10^3$ for the quarry run which eventually was exposed to the waves.
- Very little is known about scale effects related to the flow and the pore pressure in the more impervious parts of the breakwater such as the core (and the seabed if of sand). This means that for example uplift forces on concrete cappings and geotechnical aspects such as slip-circle stability and settlement cannot be properly evaluated in a scale model at the moment.

Surface tension effects

The surface tension determines the amount of entrapped air in breaking waves. As a consequence scale effects are present in scale models of forces from breaking waves and overtopping/spray. The shape (surface profile) of the waves in very small scale models is also affected.

Stive, 1985, studied the influence of air entrainment in a comparative scale model study of waves breaking on a beach. He recorded wave heights, set-up and vertical profiles of maximum seaward, maximum shoreward and time-mean horizontal velocities and found no significant deviations from the Froude scaling in a wave height range of 0.1 meter to 1.5 meter. This indeed indicates that surface tension scale effects are insignificant even in small scale models except for phenomena where a very accurate reproduction of the profile of the breaking wave is important. The most important example is shock pressures on plane solid walls. A special problem related to shock pressures is the interpretation of the recorded pressures in the model, because the air compressability is not to scale. This problem has been discussed by many researchers, see for example Lundgren, 1969, but it still remains to check model data against prototype measurements before the uncertainty related to shock pressures can be evaluated.

However, the author believes that the order of magnitude of wave pressures on wave walls found from proper scale models is correct. This opinion is based on a study of breakwater failure where damage to the concrete capping with wave walls allowed a rather accurate determination of the wave forces involved. By means of results from scale model tests, performed by DHI, in which wave pressures on the wave wall were recorded, it was possible to estimate the wave climate. This estimate was in very good agreement with the wave climate established by hindcast from meteorological observations.

Effects of mechanical properties of armour units

The relative strength of armour units is dependent on the size of the units, Burcharth, 1981. This has to be taken into account when designing and interpreting the scale models. The importance of this has been demonstrated in a number of papers by NRC, Canada, see for example Timco et al. 1983, who also developed a method of producing concrete armour units with correctly scaled mechanical properties, Timco 1981.

There are different ways of tackling this strength problem in scale models, as discussed by Burcharth, 1983, but in the case of tests with large (in prototype), complex types of unreinforced armour units the method established by NRC seems to be the best. The reliability of the method has yet to be evaluated. This can be done only by comparison with prototype measurements. A promising full scale experiment with instrumented 48 t Dolosse set up by the U.S. Army Corps of Engineers, Vicksburg, might provide very useful data for such a comparative study.

6. STOCHASTIC DESIGN PROCEDURE

It follows very clearly from the foregoing discussion that our quantitive knowledge on the loads and the structural response is limited to such an extent that design based purely on theory is not feasible. It is obvious that it will take years before we have developed a reliable design theory. Until then scale model tests are by far our most important tool.

In this rather unfortunate situation it is reasonable to think of a stochastic or probabilistic design method. However, it is often argued that a probabilistic design procedure is of little value as long as the understanding of the physics is poor. It is of course true that such a design process never gives figures in which to place high confidence as long as we cannot describe the physics. However, it is worth while to recall that the less we know, the more important it is to try to assess the reliability. The probabilistic approach is the only one which gives information on the risk of failure with due consideration to the uncertainty or scatter of the various parameters involved.

It is no excuse not to use the method because we do not know the probability density functions. As engineers we must estimate these functions, just as we estimate safety factors.

To-day's knowledge makes it of course not very easy to assess the probability functions. This is obvious from the Figures 16 and 17, which show typical failure modes and the corresponding fault tree. It is seen that not only the distribution functions for a great number of individual parameters but also the joint distribution functions for correlated parameters must be estimated.

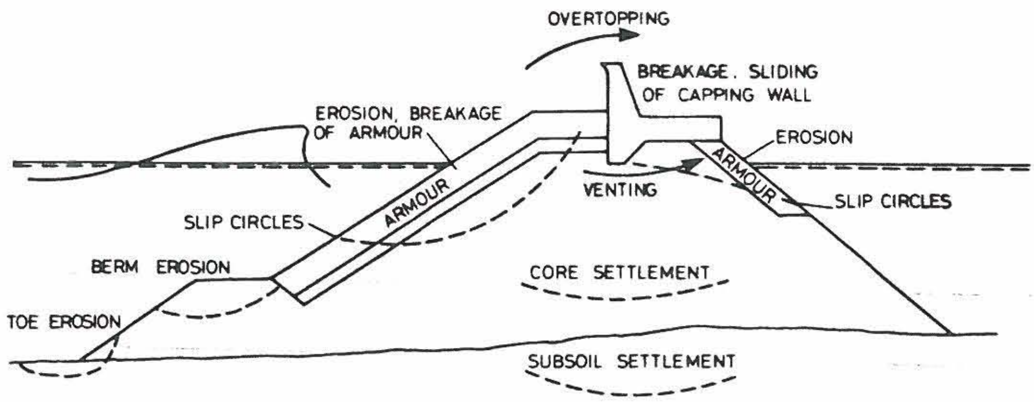


Figure 16. Failure modes of conventional rubble mound breakwater.

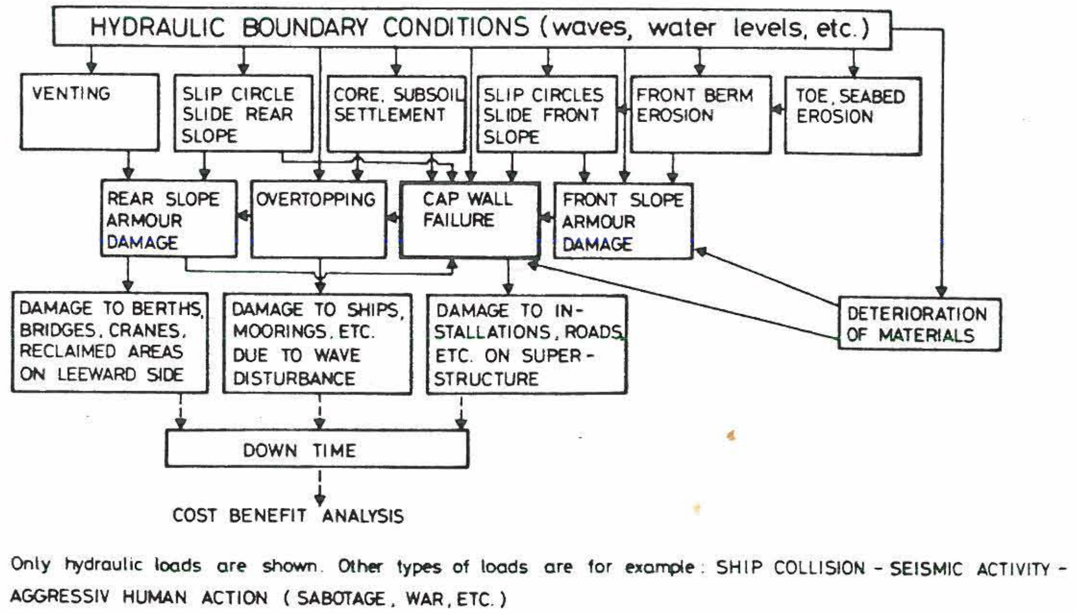


Figure 17. Simplified FAULT TREE for one section of a conventional rubble mound breakwater.

Despite all the problems one should go ahead and for a start restrict the stochastic design calculations to the vital parts of the structure only involving the most important parameters. An example is given by Nielsen et al., 1983, for the design of an armour layer. Systematical scale model experiments should be performed by capable laboratories to support this development.

In the present situation it is very important for the designer of rubble mound breakwaters to understand all the uncertainties he is up against. This might help him in developing designs which are uncomplicated in the sense of clear failure modes and load responses which are not very sensitive to exceedences in the estimated loads.

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